



Effect of embedment ratio on buried pipelines subject to combined loading

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ABSTRACT

A numerical study has been conducted to investigate the effect of soil restraint on local buckling behaviour of buried energy pipelines. The finite element software package ABAQUS/Standard has been used to develop the numerical modelling procedures. Through a parametric study, the influence of soil restraint and embedment ratio (pipeline burial depth to pipeline diameter ratio, H/D) on the pipeline peak moment capacity, critical strain and ovalization are investigated. As the H/D ratio increases, the soil failure mechanism changes from passive wedge formation to soil local failure around a pipeline. Variation of the soil failure mechanism affects the contact surface between the pipeline and surrounding soil. Therefore the magnitude and distribution of loads that can be transferred changes. Also the location of the critical section, the factor of ovalization, and the moment-strain relationship of a buried pipeline varies with increasing H/D ratio.

RÉSUMÉ

Une étude numérique a été menée pour examiner l'effet de sol de retenue sur le comportement de flambement local enterré énergie pipelines. Le finite element logiciel ABAQUS / Standard a été utilisé pour développer des la modélisation numérique des procédures. Grâce à une étude paramétrique, la influence de la retenue des sols, et l'incrustation des aux (la profondeur d'enfouissement de pipelines pipeline de diamètre, H/D) sur le pipeline moment la capacité de pointe, critique souche font l'objet d'enquêtes et d'ovalisation. Comme le H/D augmente, le mécanisme de l'évolution des sols échec de la passivité de coin formation au sol local autour de l'échec d'un pipeline. Variation de la terre mécanisme de l'échec sur la surface de contact entre le pipeline et sol environnant. Par conséquent, l'ampleur et la distribution de géotechniques de charge qui peuvent être transférées changements. Aussi la localisation des la section critique, le facteur de l'ovalisation, et le moment de souche relation d'un pipeline enfoui varie avec l'augmentation de H/D ratio.

1 INTRODUCTION

Buried pipelines cross terrain with varied environmental and geotechnical conditions. Along specific route corridors, the pipeline system may be subject to long term, large scale ground movement due to accumulated soil deformation such as subsidence, thaw settlement, frost heave, and slope movement (Bughi et al. 1996, Grivas et al. 1996, Glover et al. 2002, and Morgan et al. 2004). Under these large ground deformations, the pipeline may deform, yield, and experience local buckling mechanisms (Bruschi et al. 1995, Honegger and Nyman 2004, Kenny et al. 2007, and Mahdavi et al. 2009).

To evaluate pipeline strain capacity, physical testing and numerical modelling studies generally only consider "in-air" boundary conditions (DNV 2000, Dorey 2001, Zimmerman et al. 1995). "In-air" boundary condition means that the effect of soil restraint on the pipeline mechanical response is absent. There exists significant evidence that surrounding soil provides structural support and stability (i.e. flexible culvert design in civilian applications) and imposes different load characteristics (i.e. spatial and temporal variation in soil pressure field)

than conventional monotonic combined "in-air" loading tests (Konuk et al. 1999, Popescu et al. 2002a and 2002b, Wilkie et al. 2001).

This study is part of a doctoral research program on local buckling behaviour of buried pipelines. The objective is to study the influence of geotechnical loads and restraint on the local buckling response of buried pipelines. Two soil types, stiff clay and dense sand, were studied. Three-dimensional continuum modelling procedures were developed, using ABAQUS/Standard, and calibrated against limited physical data on the buckling response of an unpressurized buried pipeline (Mahdavi et al. 2008). A comprehensive parametric study was conducted to examine the flexural behaviour of buried pipeline, in stiff clay, under combined loading (axial force, lateral force, and internal pressure applied at a pipeline section) and a new critical strain criterion was developed (Mahdavi et al. 2009). The new critical strain criterion predicts that, for pipeline buried in stiff clay, H/D within the range of 2 to 4 does not have significant effects on the pipeline critical strain. In this study, further analyses on the effect of embedment ratio (H/D) on

bending behaviour of a buried pipeline in stiff clay was conducted and the results are presented in this paper.

2 MODEL CALIBRATION AND PARAMETRIC STUDY

The numerical model was calibrated based on available full-scale tests conducted on a buried pipeline (Mahdavi et al. 2008). A comprehensive parametric study was then conducted to investigate the effect of several parameters on local buckling response of buried pipelines in stiff clay and dense sand. The results of this parametric study in stiff clay have been presented by Mahdavi et al. (2009). The results of the parametric study in dense sand are currently under review.

The statistical design of experiments (DOE) methodology was applied to establish an efficient parametric study plan (Montgomery 2005). The main advantage of the DOE methodology is the effect of each parameter and any possible interaction between parameters on desired response can be detected through a limited number of runs (Montgomery 2005).

For the stiff clay, six parameters were considered to have the most significant effect on the local buckling response of buried pipelines. These parameters were chosen based on the results of numerical and experimental studies available in the open literature. The selected parameters are: pipeline diameter (D), pipeline diameter (D) to wall thickness (t) ratio defined as D/t , pipeline material grade, the ratio between hoop stress (σ_h) due to internal pressure and pipeline yield stress (SMYS) defined as β , the ratio between the axial force (N) and characteristic plastic axial force resistance (S_p) defined as α , and the ratio between pipeline burial depth (H) and diameter (D) defined as H/D (Table 1).

Table 1. Parametric study's factors

Parameter	Low value	High value
D (m)	0.324	0.762
D/t	51	92
Pipeline Grade	52	70
$\beta = \sigma_h / \text{SMYS}$	0	0.8
$\alpha = N / S_p$	0.25	0.56
H/D	2	4

3 NUMERICAL MODEL CHARACTERISTICS

Numerical modelling procedures were developed using finite element software ABAQUS/Standard. Figure 1 illustrates the schematic geometry of the numerical model. The pipeline length (L) and the soil dimensions in front (a), beneath (b) and behind (c) the pipeline varied proportionally with the pipeline diameter (Figure 1). To reduce the computational effort required, symmetric boundary conditions were defined at the pipeline midsection. The numerical model was calibrated using data from full-scale bending tests on an unpressurized buried pipeline at Memorial University of Newfoundland

(Konuk et al. 1999). The current study examines the effect of geotechnical loads and restraint on the local buckling response of a buried pipeline. The study does not examine mechanisms or conditions that trigger large deformation ground movement.

Lateral displacement of $1D$ to $1.5D$ was applied at a reference point (RP) defined at the pipe end. Linear, general-purpose shell reduced integration elements with finite membrane strains were used to discretize the pipeline. The soil was discretized with three dimensional solid continuum elements.

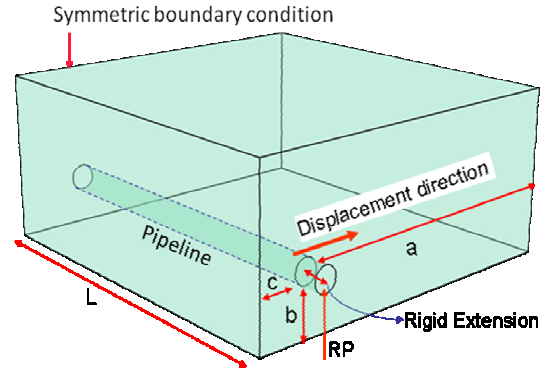


Figure 1. Numerical model geometry

The parameters of the stiff clay used, were: Undrained shear strength (C_u) = 100 kPa, Young's modulus (E_s) = 10 MPa, and Poisson ratio (ν) = 0.49 for undrained behaviour. The von Mises constitutive model implemented in ABAQUS/Standard was used to simulate stiff clay undrained behaviour (ABAQUS/Standard user manual v6.5.1). A piecewise elastoplastic constitutive model was used for the pipeline material. The stress-strain relationship of the pipeline was calculated from Ramberg-Osgood formula (Walker and Williams 1995). The pipeline Young's modulus (E) was 205 GPa.

4 NEW CRITICAL STRAIN EQUATION

Design-Expert software version 6 was used to evaluate the results of the parametric study conducted in this investigation. Response surface methodology (RSM) was used to develop the critical strain (ϵ_{crit}) equations. RSM approximates the response through regression analysis. Equation 1 was derived as the new critical strain criterion for the critical strain (ϵ_{crit}) of a buried pipeline subject to combined load state. Details of equation development have been presented in Mahdavi et al. (2009). This study has defined the critical buckling strain as the total axial compressive strain corresponding to the peak moment which is measured over a gauge length of $D/3$ along the pipeline extreme compressive fibre. A comprehensive comparison between this study's critical strain (Equation 1) for buried pipelines and several critical strain equations for "in-air" pipelines available in the open literature is under review and will be published in the near future.

$$\begin{aligned} \varepsilon_{crit} = & 0.11872 - 0.01037 * D - 2.0205E-003 * D/t - \\ & 4.51597 * SMYS/E + 0.13103 * \beta + 6.62323E-003 * \alpha + \\ & 1.18768E-005 * (D/t)^2 - 6.54164E-004 * D/t * \beta - \\ & 18.95806 * SMYS/E * \beta - 0.031132 * \beta * \alpha \end{aligned} \quad [1]$$

According to statistical analyses, D , D/t , $SMYS/E$, β , $(D/t)^2$, and the interaction effects of D/t and β , $SMYS/E$ and β , and finally β and α were recognized as the significant model terms. Also the statistical analysis indicated that, with respect to critical strain, H/D over the range of 2 to 4 was not as significant as the other parameters for pipes in stiff clay. Sufficient soil resistance was available to initiate failure of the pipe rather than the soil system.

5 EMBEDMENT RATIO EFFECT

The significance of embedment ratio (H/D) on the pipeline strain response was further investigated over a wider range of H/D ratios from the parametric study defined in Table 1. Eight cases were examined to study the effect of H/D on bending behaviour of buried pipeline (Table 2). The first 4 cases examined an unpressurized small diameter pipeline with the H/D varying from 1 to 8, whereas the second group examined a larger diameter, pressurized pipeline with H/D varying from 2 to 8.

H/D affects the interaction forces at the pipeline/soil interface, soil normal stress and soil failure mechanisms. As a result pipeline bending moment capacity, curvature, ovalization, and critical strain can be affected by H/D variation depending on pipeline/soil characteristics and loading conditions. These effects are discussed in more detail later in Section 5.1.

Table 2. Characteristics of conducted numerical analyses

Analysis Case	D (m)	D/t	Pipeline Grade	β	α	H/D	L/D
1	0.324	51	52	0	0.25	1	9
2	0.324	51	52	0	0.25	2	9
3	0.324	51	52	0	0.25	4	9
4	0.324	51	52	0	0.25	8	9
5	0.543	72	60	0.4	0.405	2	9
6	0.543	72	60	0.4	0.405	3	9
7	0.543	72	60	0.4	0.405	4	9
8	0.543	72	60	0.4	0.405	8	9

5.1 Analysis Cases 1 to 4: Results and Discussion

Table 3 summarizes the results of the first 4 analyses in terms of normalized critical strain and normalized maximum bending moment, all corresponded to the peak bending moment increment. The critical strain is normalized with the critical buckling strain equation as defined in Canadian Standard Association Z662 (CSA 2003). The bending moment at critical section (M_{max}) is

normalized by a pipeline maximum bending moment capacity (M_o) as defined in DNV OS-F101 (2000).

Table 3. Results for analysis cases 1 to 4

Analysis number	H/D	$\varepsilon_{crit}/\varepsilon_{CSA}$	M/ M_o
1	1	1.85	0.74
2	2	4.67	0.65
3	4	4.53	0.74
4	8	4.42	0.66

As presented in Table 3 the current study's critical strains are larger (approximately two to four times) than CSA critical strain. The surrounding soil has a restraining effect, which increases the pipeline bending resistance, when the pipeline is subjected to large ground deformations.

Figure 2 compares the bending moment-critical strain relationship of the first 4 analyses. The critical strain of each analysis is indicated on the figure. As H/D increases from 1 to 2, the critical strain significantly increases. As H/D increases from 2 to 4 the critical strain shows a minor change (3%) which is consistent with the Eq. 1 prediction. Increasing H/D from 4 to 8 does not show significant changes in critical strain.

Figure 2 shows that as the H/D ratio increases, the strain, prior to the sudden reduction in load carrying capacity of the pipeline (indicated with a small arrow on each diagram), also increases. For the analysis case with the H/D ratio of 8, the numerical solution encountered convergence problems before the bending moment reached to the sudden drop point. Increasing the H/D ratio, increases the soil pressure and restraint, changes the soil failure mechanism, and reduces pipeline sectional deformation mechanisms. These effects tend to promote structural stability of the pipeline system with increasing curvature or strain.

Figure 3 compares the variation of factor of ovalization (f), corresponding to the peak moment increment, along the pipeline for analysis cases 1 to 4. The ovalization factor was calculated from Equation 2 in which D_{max} and D_{min} are the maximum and minimum measured diameter respectively of the pipeline. The variation of maximum ovalization factor with H/D is consistent with the variation of critical strain with H/D . This is expected due to the fact that the factor of ovalization approximates a pipeline section out of roundness.

Pipeline ovalization occurs due to bending and can be a precursor to local buckling mechanisms. The soil restraint limits this mode of pipeline response. For shallow buried pipelines it appears that soil loads are reduced and thus pipeline ovalization response is moderated.

$$f = \frac{D_{max} - D_{min}}{D_{max} + D_{min}} \quad [2]$$

Figure 4 presents pipeline lateral force variation with local strain for analysis cases 1 to 4. The pipeline lateral force is the force imposed at the reference point (RP in Figure 1) in order to mobilize the pipeline lateral displacement. The lateral force is output by the program as a reaction force.

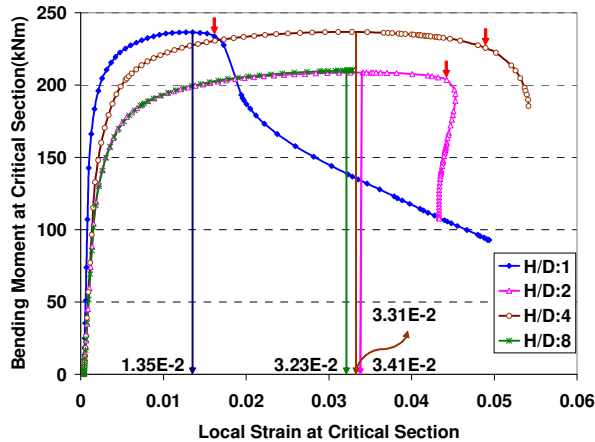


Figure 2. Bending moment variation with local strain at critical section for analysis cases 1 to 4

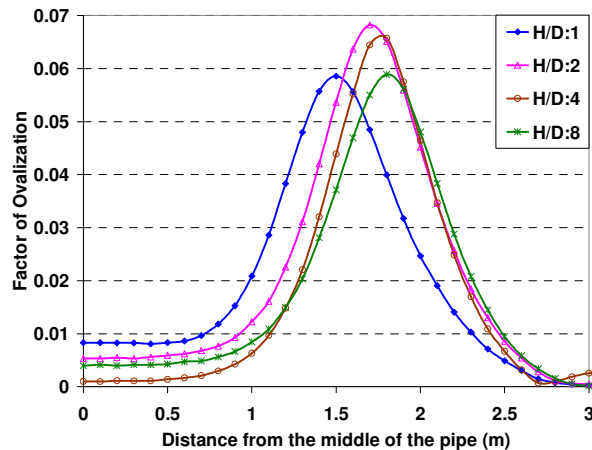


Figure 3. Factor of ovalization along the pipeline for analysis cases 1 to 4

As H/D increases from 1 to 2 the peak lateral force increases significantly. When H/D increases from 2 to 8 the peak lateral force further slightly increases. The lateral force is in equilibrium with lateral compressive soil forces imposed on a pipeline, in front (near the pipeline end) and at the rear side (near the middle of the pipe).

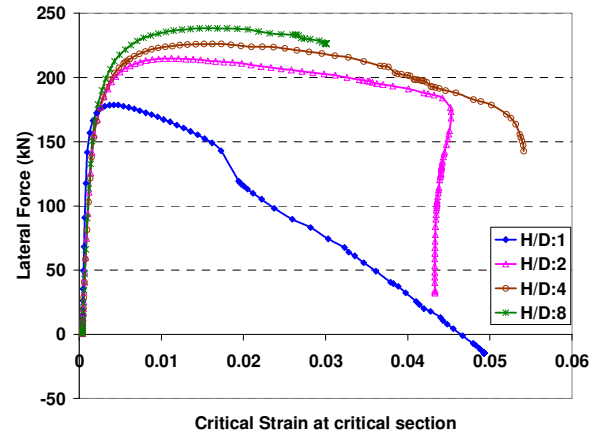


Figure 4. Lateral force versus critical strain for analysis cases 1 to 4

As the lateral pipeline displacement at RP increases, the size of the soil failure zone gradually increases and mobilizes additional soil resistance. Soil in front of the pipe yields, this results in decreased soil stiffness and larger pipeline deflections. The lateral force increases to a maximum where strain localization initiates at a critical section of the pipe (Figure 4). Increasing lateral pipeline deflection results in the development of a plastic hinge. It should be noted that variations in the pipe-soil interactive behaviour depends on whether failure occurs first in the soil or pipeline. This behaviour is a function of the pipeline material properties, loading conditions and geometry, pipeline burial depth, soil characteristics, and boundary conditions.

Figure 5 illustrates the plastic strain distribution in soil, corresponding to the peak moment, for analysis cases 1 to 4. According to this figure as H/D increases the soil failure mechanism changes. For a shallower H/D of 1 and 2 the failure mechanism is the formation of a passive wedge in front of the pipe. For H/D = 1 the wedge is fully shaped and causes significant soil surface heave (Figure 5 (a)). A passive wedge is also formed and causes soil surface heave for H/D of 2 but not as large as for H/D of 1 (Figure 5 (b)). In both cases the wedge is formed in front of the pipe and extended to the surface. As H/D increases to 4 and 8 the soil failure mechanism changes. Instead of a passive wedge in front of the pipe, soil fails locally and flows around the pipe without noticeable soil surface heave (Figure 5 (c) and (d)). Similar soil failure mechanism variation with H/D was observed in a study conducted by Philips et al. (2004). Variation of the soil failure mechanism affects the stress distribution between the pipeline and surrounding soil.

The global curvature and buckling location also vary as H/D increases. Figure 6 illustrates global curvature variation with local strain for analysis cases 1 to 4. Global curvature is defined as the ratio of bending rotation at the reference point to the pipeline length.

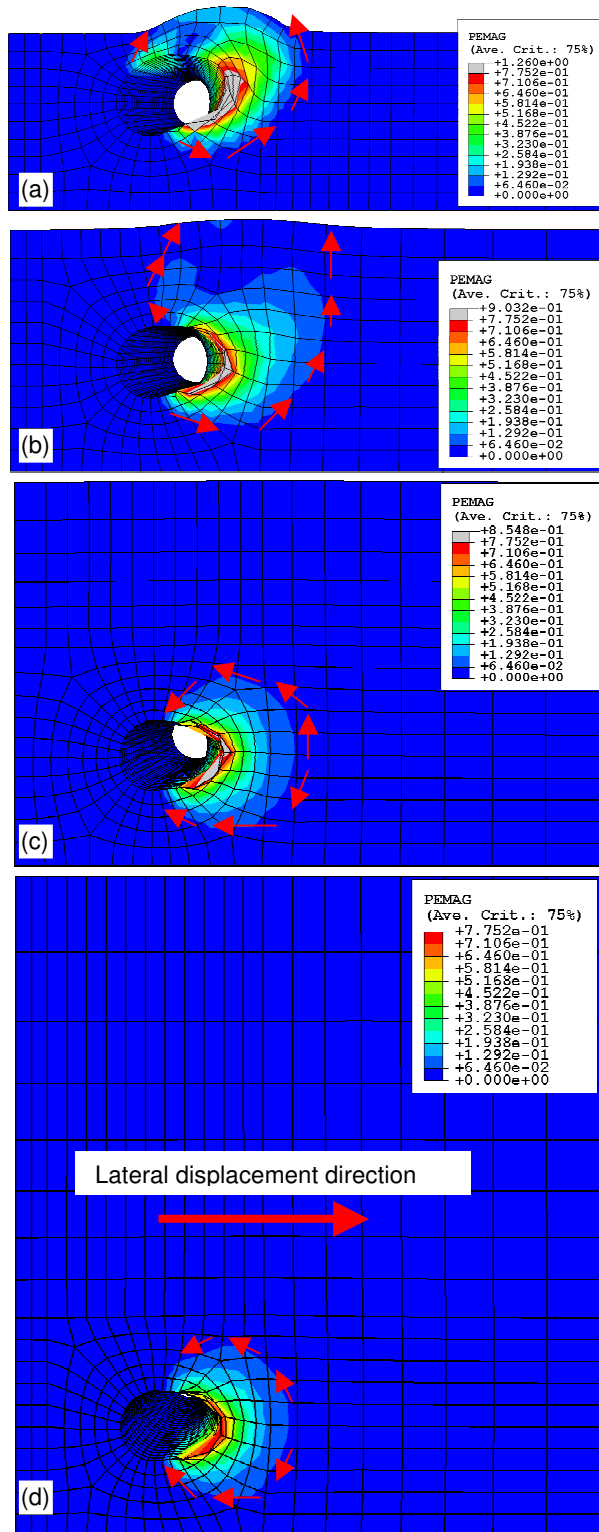


Figure 5. Soil magnitude of plastic strain distribution – (a) to (d) represent analysis cases 1 to 4, respectively

The bending moment along the pipeline is a difference between moment due to active forces (i.e.

pipeline lateral force and axial force) and moments due to soil resistance. As explained before when H/D increases the pipeline lateral force, location of buckling, global curvature, soil lateral stress, soil failure mechanism and therefore the contact surface between a pipeline and surrounding soil change. As a result the bending moment may increase or decrease which explains the non-uniform bending moment variation with H/D presented in Table 3.

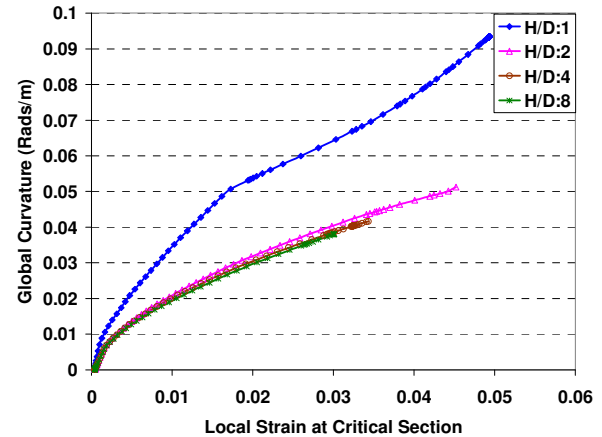


Figure 6. Global curvature versus local strain for analysis cases 1 to 4

5.2 Analysis Cases 5 to 8: Results and Discussion

Table 4 presents the results of analysis cases 5 to 8 (pressurized, medium diameter, X70 pipeline). Similar to the previous section, the current study's critical strains, presented in Table 4, are larger (approximately 4 to 6 times) than limit strain predicted by Canadian Standard Association (CSA 2003). According to Table 4 as H/D increases from 2 to 4 the critical strain increases up to 28% however as H/D increases to 8 the critical strain decreases up to 13%.

Table 4. Results for analysis cases 5 to 8

Analysis Case	H/D	$\epsilon_{crit}/\epsilon_{CSA}$	M/M_o
5	2	4.92	0.55
6	3	5.34	0.56
7	4	6.33	0.56
8	8	5.60	0.55

Figure 7 shows bending moment variation with local strain for analysis cases 5 to 8. According to this figure the peak bending moment does not change as H/D changes. The strain corresponding to the beginning of the sharp vertical drop in moment (indicated by small arrows) increases as H/D increases from 2 to 4.

Figure 8 illustrates pipeline lateral force versus local strain for analysis cases 5 to 8. The lateral force increases as H/D increases. As the H/D ratio increases larger soil pressure are mobilized in front of the pipeline

therefore a larger lateral force is required to displace the pipe.

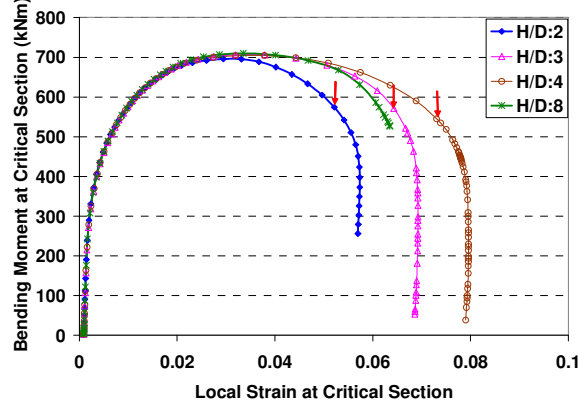


Figure 7. Bending moment versus local strain for analysis cases 5 to 8

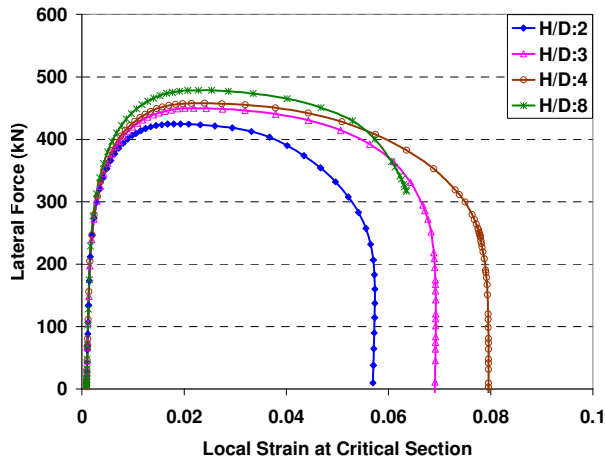


Figure 8. Lateral force versus local strain for analysis cases 5 to 8

Figure 9 illustrates variation of the ovalization factor along the pipeline, corresponding to the peak moment increment, for analysis cases 5 to 8. Two sections, located approximately 1.2 m and 1.7 m away from the pipe end (Figure 9) have a higher factor of ovalization which indicates the formation of two bulging sections along the pipeline. Variation of the maximum factor of ovalization with H/D ratio is consistent with variation of critical strain with H/D ratio. A similar collapsing mode (a series of wrinkles) for a buried pressurized pipeline was reported in a separate study conducted by Popescu et al. (2002a).

Figure 10 compares global curvature versus local strain for analysis cases 5 to 8. As H/D ratio changes from 2 to 8 the global curvature does not show significant changes.

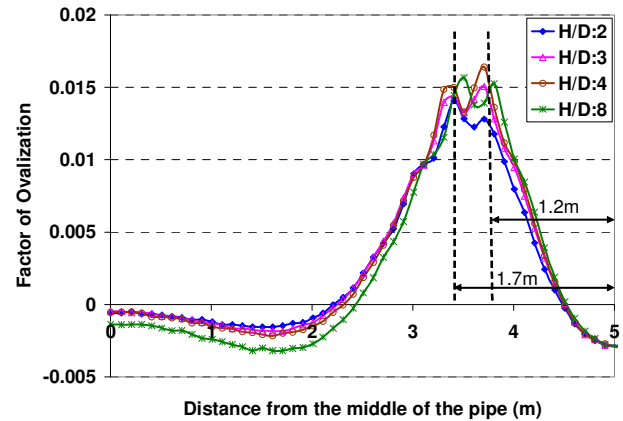


Figure 9. Factor of ovalization along the pipeline for analysis cases 5 to 8

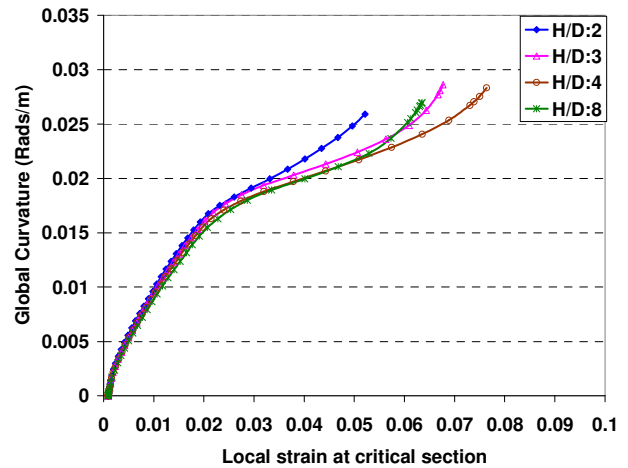


Figure 10. Global curvature versus local strain for analysis cases 5 to 8

6 CONCLUSION

Further examination of the effects of embedment ratio (H/D) on bending behaviour of a pipeline over the H/D range of 1 to 8 was studied through the numerical approach. In this paper the results of 8 analyses including 4 unpressurized pipelines with H/D ranging from 1 to 8 and 4 pressurized pipelines with H/D ranging from 2 to 8 were presented.

As H/D increases the soil resistance against a pipeline lateral displacement increases. For a shallower H/D of 1 and 2 the surrounding soil fails in the form of a passive wedge initiating in front of the pipe and extending toward the surface which can be characterised with noticeable soil surface heave. As the burial depth becomes deeper the soil fails locally and flows around the pipe. Variation of the soil failure mechanism affects the contact surface between a pipeline and soil. Therefore the amount of stress that can be transferred changes. Also the location of the critical section and the global curvature vary which can change the amount of bending moment along the pipeline.

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